New Hanger Design Approach of Tied- Arch Bridge to Enhance Its Robustness

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Abstract: As the crucial components among the tied-arch bridge, the local failure of hangers may trigger a progressive collapse through the entire tied-arch bridge. However, the current design guidance as regards hangers still lacks consideration of structure robustness under an extreme hazard. To improve the structural robustness of tied-arch bridge under extreme conditions, a new hanger design method is proposed, which is termed as asymmetric parallel double-hanger system. Based on Miner's linear cumulative damage law, an analysis on the fatigue life of the double-hanger system was conducted to verify the feasibility of the proposal, and then a dynamic time-history analysis was employed to simulate the transitory fracture impact due to one or more hangers fracturing. According to the simulation results, the structural robustness is greatly enhanced with asymmetric parallel-double hanger system design, when compared with single hanger system design. When one or more hangers reveal local damage, it will not trigger a progress failure to the whole structure in particular. Several practical suggestions of bridge system's load-carrying capacity are also put forward for the future arch bridge design at the end of this paper.

Keywords: Tied-arch bridge; Alternative load path; Double hanger system; Sudden removal; Fatigue life.

1. Introduction

Structural systems optimized to meet member design criteria as specified in current design standards and specifications may not provide sufficient levels of robustness to withstand a possible local failure under an unforeseen extreme event. In fact, local failure in one structural element may result in the failure of another. The chain reaction of failures that progress throughout the structure will cause a level of damage disproportionate to the initial damage, even a catastrophic collapse of the whole structure. (ASCE, 2002; Ellingwood and Dusenberry, 2005). Such progressive collapse occurs, because a sudden local change in structural geometry due to the loss of load-carrying members will result in extra dynamic force in surrounding elements, which may exceed the bearing capacities of them (Buscemi and Marjanishvili, 2005).

Catastrophic events, such as the collapse of the Alfred P. Murrah Federal Building in Oklahoma City in 1995, the I-35W Mississippi River Bridge in Minnesota in 2007 and the I-5 Mount Vernon WA Bridge in 2013, have given an alarm about the structural survivability after an initial local failure. Meanwhile, the lack provisions of structural integrity or robustness in current design codes have got more attention from structural engineering community. Some efforts have been contributed, for instance, by the US General Service Administration and US Department of Defense, which have announced the guidelines of progressive collapse assessment method (GSA, 2003; US DoD, 2005). Furthermore, enhancing structural robustness in design codes has also been considered in other countries (Pearson and Delatte, 2005).

As the reliable structural damage detection is still a big challenge, a rational design approach should be a threat-independent method, by which it could avoid designing for an extreme event with specific action magnitude that may exceed the normal loading condition during the service life. This can be achieved through structural robustness, which is defined as “the ability of a structure to withstand events like fire, explosion, impact or consequence of human error, without being damaged to an extent disproportionate to the original cause”, according to EN1991-1-7 Euro code 1 (BSI, 2006). According to Euro code 1, the local damage is acceptable only if the following two principles can be guaranteed. The first is that the local damage will not endanger the whole structure. The second is that the overall load-carrying is maintained during an appropriate length of time to allow the necessary emergency measures to be taken (Gulvanessian and Vrouwenvelder , 2006).

According to the mentioned design principle, the alternative load path design method is the pragmatic option for structure engineers, instead of tying force method (Starossek,2007) and specific load resistance method (Paramasivam, 2008) due to their limitations in real applications (Byfield, 2004; Byfield and Paramasivam, 2007; Ellingwood et al., 2007).

By the alternative load path design method, the structure is designed so that a new load path could be developed to pass through the local failure zone. The alternative load path relies on the ‘robustness’ of the structure (Agarwal, 2011), which is achieved through continuity and ductility of members to redistribute force following localized damage. The more important point from this design method is to direct the designer’s attention towards the behavior of the structure after some damage has occurred (Starossek, 2007; Morison et al., 2014).

The basic procedure of the alternative load path analysis, given by ASCE, US GSA and US DoD, is analyzing the damaged structure with a specific loading to check if the initial damage propagates. The damage is introduced by notional removal of one primary load-bearing member at a time. Four analytical approaches for alternative load path analysis have been approved by the US GSA and the US DoD, which are linear static, non linear static, linear dynamic and non-linear dynamic analysis (ASCE, 2002; GSA, 2003; US DoD, 2005). However, these existing guidelines were developed for buildings and may not be suitable for bridges, because of the differences in their topologies, configurations and load conditions. Therefore, much more efforts are desired for the development of bridge design guidelines. (Starossek, 2007; Giorgio et al, 2013).

The through tied-arch bridges have been widely constructed in China since 1990s. However, there is still a big gap between the research outcome and the mature design theory. Unexpected accidents, i.e. structure collapse of tied-arch bridge, cannot be ignored anymore (Chen and Wang, 2009), which are listed partially in Table 1.

Among all the listed bridges in Table 1, hanger fracture and overload is responsible for most bridges' collapse, except Qijiang Rainbow Bridge in Chongqing city. According to Chen and Wang (2009), the hanger fracture is generally the result of hanger stand corrosion or anchor head corrosion, protective layer damage or short hanger damage, or anchor head joint damage.

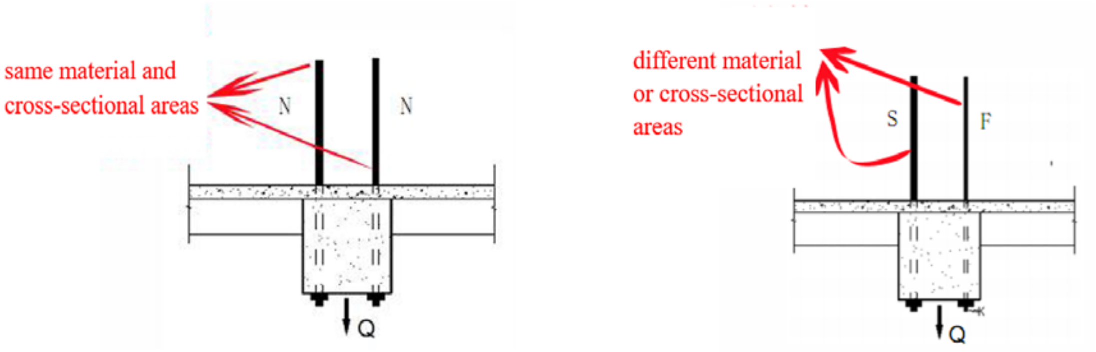
Table 1-Through tied-arch bridge accident in China since 1999

Bridge Name	Collapse Date	Collapse cause
Qijiang rainbow bridge in Chongqing	Jan. 11,1999	Low construction quality
Yibin South Gate Bridge in Sichuan	Nov. 7,2001	Hanger fracture and overload
Changzhou Canal Bridge in Jiangsu	May 14,2007	Hanger fracture
Yuping Mountain Bridge in Fujian	Jan. 11,2010	Hanger fracture and overload
Peacock River Bridge in Xinjiang	Apr. 12,2011	Hanger fracture and overload
Tongyu River Bridge in Jiangsu	Jul. 11,2011	Hanger fracture
Wuyishan mansion Bridge in Fujian	Jul. 11,2011	Hanger fracture
Luoguo Jinsha River Bridge in Sichuan	Dec.10,2012	Hanger fracture

Due to its vulnerability to fatigue phenomena, hangers can be treated as one of the most significant components in a through-arch bridge system. Local damage at a hanger may lead to subsequent damage of various components in the vicinity or even progressive collapse of the whole bridge. Hong and Khudeira introduced an innovative application of a new design technique by providing a pair of structural strands at each hanger location, which is the way for advancing part of the load-path redundancy (Hong and Khudeira, 2014). Instead of using two identical hangers in the conventional design of double-hanger system, Jiang et al (2013) suggested to use two different hangers to increase the safety factor of the members in the vicinity of local damage, in order to improve the robustness of the through-arch bridge. However, few efforts are devoted to enhance the robustness of tied-arch bridge by improving hanger design approach. Hence, for attenuating the probability of the progressive collapse, this paper put forward a new design concept for tied-arch bridge hangers, which is named as asymmetric parallel double-hanger system. Its mechanism will be analyzed to evaluate its feasibility for enhancing the bridge's robustness.

2. Introduction of Asymmetric Parallel Double Hanger System

The double-hanger anchorage (Fig.1a) is often used with its higher safety and more convenience of hanger replacement, when compared with the single-hanger anchorage (Hong, 2014). The two hangers at the same anchorage are generally designed with the same material and cross-section area. Theoretically, the probability of fracture of those two hangers is the same because they are exposed to the same loading circumstance. In this case, this design method has two important limitations. There is a great uncertainty regarding which of the two hangers is the first one to fail, and the resulting impact due to the sudden fracture of one hanger would cause another hanger at the same anchorage fracturing promptly. Furthermore, it would trig a chain reaction of progressive collapse of tied-arch bridge. Therefore, the current design method cannot improve the safety and the convenience of hanger replacement.



a) Symmetrical parallel double hanger system b) Asymmetric parallel double hanger system

Fig. 1 Two systems of parallel double-hanger

According to the alternative load path, one of hangers at the same anchorage has to be designed with a different parameter from another, for ensuring that the two hangers could not fracture simultaneously. For that purpose, a new design concept, which is named as asymmetric parallel double-hanger system, is proposed firstly in this paper, as shown in Fig.1b. Analysis on its function mechanism is then focused in this paper for improving the robustness of tied-arch bridge.

According to the fatigue S - N curve for steel strands in Fig.2, the hanger fatigue life is quite sensitive to the stress level. For instance, two hangers will have an obviously different fatigue life, when their stress difference increases to a certain proportion, i.e. 10% (Soltani et al, 2012). This is the prerequisite to use the asymmetric parallel double hanger system to limit the local damage of tied-arch bridge.

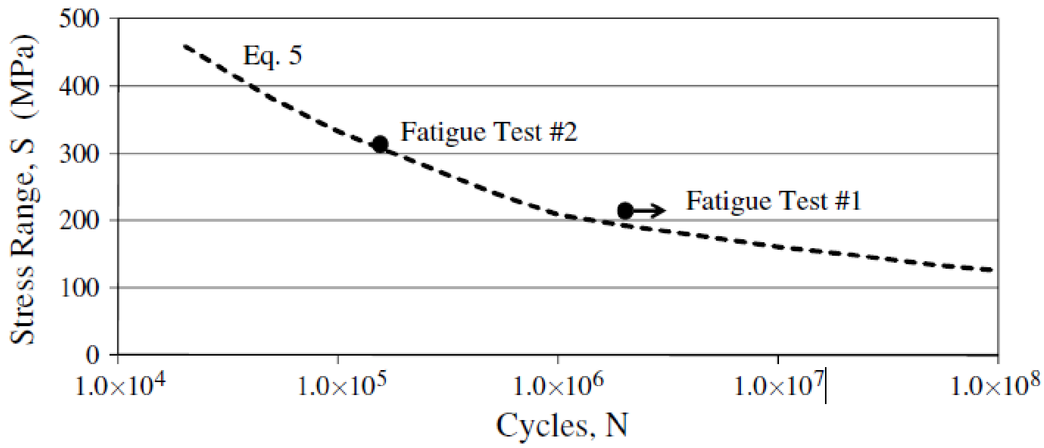


Fig. 2 Predicted and experimental S-N data

The asymmetric parallel double-hanger system has two hangers with different cross-sectional areas, as shown in Fig.1b. One of them with smaller cross-sectional area is defined as the failure hanger, referred to as *F* hanger, provided that it is the first fracturing hanger in case of local damage. Another one with a larger cross-sectional area is defined as the safety hanger, referred to as *S* hanger, as shown in Fig.1b, provided that the hanger could not fracture simultaneously in case of local damage. This paper only considers the damage caused by fatigue loads, and the material defects and manufacturing defects are not considered. Based on the mentioned fatigue life theory, the fatigue life difference between two hangers could occur due to the cross-section area difference.

In this case, once the *F* hanger fractures, the *S* hanger will temporarily endure all loads. For this purpose, two design objectives need to be reached as follows. Firstly, the fracture of the failure hanger will not cause the fracture of the safety hanger immediately. Secondly, after the failure hanger fractures, the rest of the hanger system, which stands all the structural force, should work properly for a certain period, to provide enough time for hanger replacement.

3. Analysis on the fatigue life difference of asymmetric parallel double hangers

A through-type tied-arch bridge is employed here to study the function mechanism of the proposed design method. The Luoguo Arch Bridge is located at Yalong River estuary near Yinjiang Town, Panzhihua City, Sichuan Province of China. The bridge is a half-through tied-arch bridge with a 160 m main span, floating deck system and reinforced concrete arch rib. The longitudinal beams are the structure of the floating deck system of this bridge, composed by a number of simply supported longitudinal segments. The segments within a range of central span arch are supported by the transverse beams, while others are supported by transverse caps. This bridge was originally designed with a vertical single hanger system.

In order to assess the feasibility of the proposed method for structure robustness enhancement, Luoguo Arch Bridge will be redesigned by the author, with the asymmetric double-hanger system in this paper. Figure 3 shows the geometry overview of the redesigned bridge. There are 13 pairs of hangers in the north side of the bridge deck, which are numbered as 1-13 from west to east, while another 13 pairs of hangers in the south side follow the same rule for convenience. The two hangers, sharing the same anchorage, are termed as *a* and *b* for the south arch and *a*' and *b*' for the north arch (see Fig. 4).

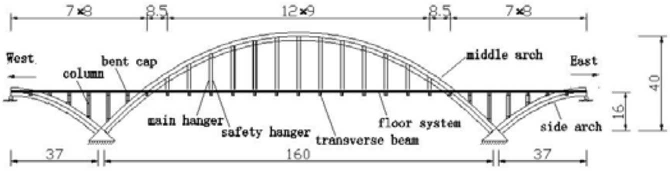
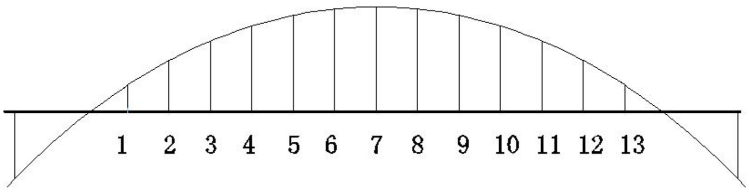
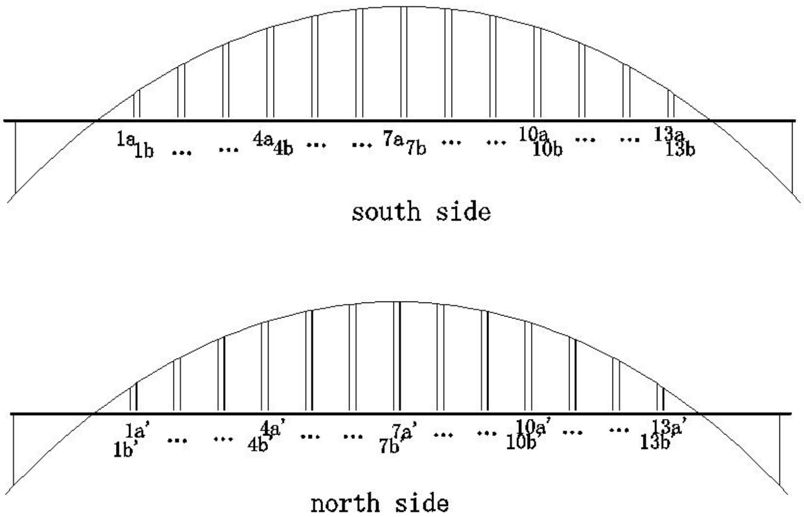


Fig. 3 Overview of the redesigned bridge with asymmetric parallel double-hanger system (Unit： m)



a) Single hanger system



b) Asymmetric parallel double-hangers system

Fig.4 Hangers numbering rule for tied arch bridge

For the asymmetric parallel double-hanger system as shown in the Fig.4b, Number 1a to 13a represent the failure hangers in the south, while the corresponding number, 1b to 13b, stand for safety hangers, and the same pattern is employed in the north arch.

It is assumed that one of two hangers (hanger a) bears most of load, acting as the failure element, and the other one (hanger b) at the same anchorage could bear a partial load, acting as a safe element, therefore a fail-safe unit (FSU) is formed. The stress of failure element need to reach about 10% more than that of safe element, so the use of the cross-section area of failure hanger is 0.905 times that of related safety hanger ,such as $A_{1a} = 0.905A_{1b}$, while their gross area is the same as that of the single hanger in traditional design system, e.g. $A_{1a}+A_{1b}$ in Fig. 4b is equal to A_1 in

Figure 4a. The way to achieve the stress difference between two hangers is that an elastic cushion with a smaller stiffness is mounted between the anchorage at the lower end of hanger b and the bearing surface of transverse beam. The maximum elastic resistance is equal to about 10% of the design internal force of the conventional parallel double hanger, and the maximum compressible height is equal to 10% of the elastic elongation of hangers. Be clear to see Fig.1, the FSU element is the same with the conventional parallel double suspender as its shape, but is not the same as the design theory, and also with a variance in structure pattern and parameters, their structure function is not the same at all.

3.1 Introduction to Palmgren-Miner linear cumulative damage law

The vehicle loads, which cause structural fatigue damage, are assumed as variable amplitude cyclic loading, and then they are treated as a combination of a series of unvaried amplitude cyclic loading (Fatemi and Yang, 1998). The Palmgren-Miner linear cumulative damage law shows that when a structure endures a series of unvaried amplitude cyclic stresses σ_{i}, its corresponding fatigue life can be assumed as N_i, then the fatigue life N of the hanger under variable amplitude cyclic stress can be calculated by the formula as follow (Fatemi and Yang,1998):

$$\mathrm{N}=\frac{1}{\sum_{\mathrm{i}=1}^{k}(\frac{n_i^T}{N_i})}=\frac{1}{\frac{n_1^T}{N_1}+\frac{n_2^T}{N_2}+...+\frac{n_k^T}{N_k}} \tag{1}$$

Where, N_i is the fatigue life of hanger under unvaried amplitude stress σ_{i}, calculated by a specific S-N curve, niT is the cycle number under unvaried amplitude stress for each hanger, which can be obtained from the fatigue loading spectrum of the traffic flow data of vehicle. The specific S-N curve is proposed by the University of Texas in the United States (Essliger, 1992), and calculated by the following formulas (2).

$$\lg N_i = 14.36 - 3.5 \lg \Delta\sigma_i, \quad \Delta\sigma_i \geq 200 \tag{2a}$$

$$\lg N_i = 37.187 - 13.423 \lg \Delta\sigma_i, \quad \Delta\sigma_i < 200 \tag{2b}$$

Where, $\Delta\sigma_i$ is the stress range of the hanger under typical vehicle loading.

3.2 Fatigue life prediction of double hangers of tied arch bridge

The fatigue loading model of this bridge, which is taken from a related literature to calculate the fatigue life of hangers (Xia, et al 2014), has 4 kinds of fatigue check-calculation vehicle loading, which are labeled as M1, M2, M3 and M4 respectively. Due to its symmetry, the anchorages No.1 to No.7 are selected for further study. Based on the result calculated with FEM (see Fig. 5), their stress amplitude under typical vehicle loading is given in Table 2.

Based on the stress amplitude of hangers mentioned above, the fatigue lives of all hangers can be predicted, by using the Palmgren-Miner linear cumulative damage law and finite element analyst, which is shown in Table 3.

Table 2 Stress amplitude of hangers for double-hanger system (Unit: MPa)

Hanger number \ load case		M1	M2	M3	M4
1	a	48.9	132	178	184
	b	43.0	116	156	161
2	a	47.8	129	175	180
	b	43.6	118	159	164
3	a	47.5	128	174	179
	b	43.7	118	160	165
4	a	47.4	128	173	179
	b	43.7	118	160	165
5	a	47.3	128	173	179
	b	43.7	118	160	165
6	a	47.2	128	173	178
	b	43.7	118	160	165
7	a	47.1	127	172	178

	b	43.7	118	160	165

It is obvious that the fatigue life of safety hanger, represented as *b*, is significantly longer than that of failure hanger, labeled as *a*, in the same anchorage. This can also demonstrate that the different stress amplitude in two hangers could lead to their different fatigue lives. Therefore, the failure hanger (hanger a) should fail first, instead of simultaneously fracturing with safety hanger (hanger b).

When compared with conventional design method, i.e. the single hanger system, this double-hanger system has two major contributions as follows. First, a slight variance in cross sections of two hangers could induce a remarkable difference in their fatigue lives, as the fatigue lives of the safety hanger can be extended as 3 times as that of the failure hanger in this new system, with just 10% variance in their cross-section areas. Second, the hanger's effective live could reduce significantly if corrosion on steel strands occurs, as the fatigue life of hanger *a* with smaller cross section is much shorter than that of hanger *b*.

Table 3 The fatigue lives of all hangers

Hanger number	Fatigue life /year	
	a	b
1	22.05	126.61
2	28.02	97.58
3	30.49	93.53
4	31.70	93.52
5	32.70	93.52
6	33.22	92.74
7	33.73	91.94

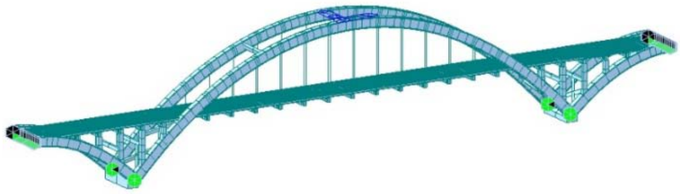
4. Dynamic analysis on failure safety for hangers of tied arch bridge

A real tied-arch bridge is considered with two types of hanger arrangement, the single hanger system and the asymmetric parallel double-hanger system. In both hanger systems, if a hanger fractures suddenly, the dynamic stress in adjacent hangers will increase dramatically, and will oscillate for a while before getting the stable value of the new increased static stress. If this maximum stress in the adjacent hanger due to transient impact effects is high enough to fracture this hanger, it may cause progress failure of the whole structure. To guarantee the bridge's robustness, the impact effect, caused by sudden hanger fracturing on components in the vicinity and the remaining structure, should be first evaluated in detail. To simulate the sudden fracturing of a hanger, the fractured member is removed from the model and replaced by a set of internal dynamic loading to the remaining structure. The set of applied load is modeled by using a steady internal force in service there, which is then assumed to linearly decrease to zero within a duration δt described in a related reference(Jiang, et al ,2013).

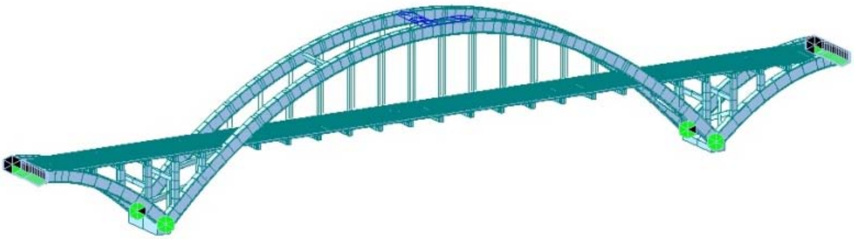
In the next three subsections, the dynamic analysis of new designed Luoguo Tied-arch Bridge with asymmetric parallel double-hanger system will be discussed and compared with the original one, which is designed with single hanger system.

4.1 Finite element analysis model

The finite element model of the arch bridge with single hanger system has 2935 nodes and 4510 elements, as shown in Fig.5a, while the other one with the asymmetrical parallel double- hanger system has 2987 nodes and 4536 elements, referring to Fig.5b. In these two models, the arch foot is restricted to 6 degrees of freedom, and the arch crown is restricted to the vertical degree of freedom. The vehicle live load and dead load are taken into account in this paper, in which the vehicle live load is arranged in a form of concentrated load P according to the most unfavorable position.



a) Single hanger system



b) Double hanger system

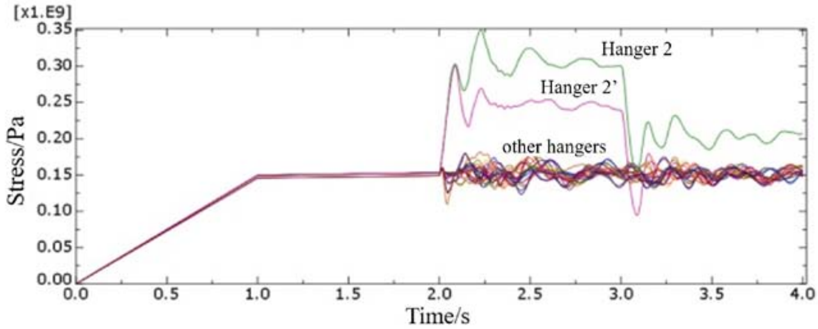
Fig. 5 Finite element model of the whole bridge

4.2 Maximum stress of remaining hangers after one short hanger fracturing

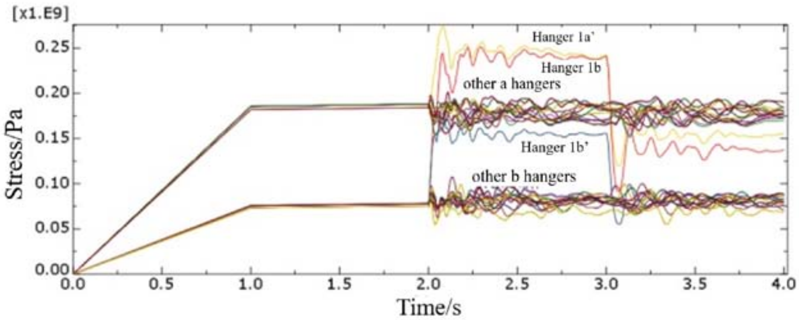
Many accidents reveal that the fracture of hangers began with the shorter hanger near the end of arch (Kondoh, et al, 2001). It is clear that the shortest hanger 1a has the maximum stress amplitude under 4 types of fatigue vehicle loading, as shown in Table 2, and the same hanger has the shortest fatigue life in Table 3. Therefore, it can be assumed that hanger 1a will fracture first in the double-hanger system, same as the single hanger system.

Assuming that the average duration of hanger fracture δt ranges from 0.01s to 1s(Jiang, et al ,2013) , δt is taken as 0.01s in this paper, for considering the most negative condition. The dynamic analysis of sudden fracture of hanger 1 in the singer hanger system is referred as case 1, while the sudden fracture of hanger 1a in the parallel double-hanger system is termed as case 2.

Figure 6a shows the tensile stress variation of remaining hangers in case 1, while in Figure 6b the same information is depicted for case 2.



a) For single hanger system (Case 1)



b) For asymmetrical parallel double-hanger system (Case 2)

Fig.6 Tensile stress variation of the remaining hangers due to one hanger sudden fracture

It can be seen from Figure 6 that:

1) In single hanger system (Case 1), after hanger 1 fractured, the tensile stress in adjacent hangers, i.e. hanger 2 and 2’, have a obvious increase, while relatively slight variations can be observed among other hangers. The maximum stress variation is 200MPa in hanger 2, increasing the total stress about 133% when compared with its static loading stress, 150MPa. Therefore, hanger 2 is most likely to be damaged.

2) In the asymmetric parallel double hanger system (Case 2), if hanger 1a at the south arch suddenly fractured, the maximum stress response would be noticed in hanger 1a’ at the north arch, while hanger 1b at the south arch would also suffer a high stress, just slightly lower than hanger 1a’. The maximum stress amplification is 275MPa in hanger 1a’, increasing about 53% when compared with the static loading stress, 180MPa. Because the design tensile strength of

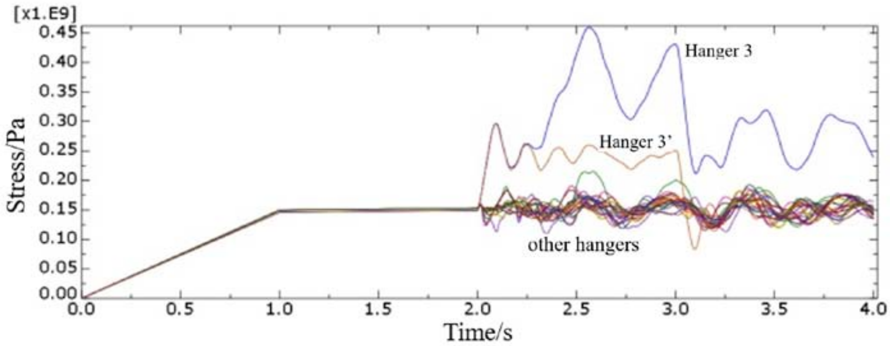
high-strength steel strands of hangers is 1130MPa, the safety factor of 1a’ reaches to 4.11, which is larger than the lower limit of 2.5 proposed by the Design Rules for Highway Cable-Stayed Bridge of China (MTPRC, 1996). Therefore, the fact shows that if tied-arch bridge is designed with the asymmetric parallel double-hanger system, the fracture of failure hanger does not trigger a progress failure of safety hanger at the same anchorage. Because an alternative load path is formed by the safety hanger in the vicinity of local damage zone after the failure hanger fracturing, then the robustness of the whole structure is enhanced to a great extent.

3) In the case of a hanger sudden fracturing at the end anchorage, the maximum impact stress in hanger 2 under the single hanger system is larger than that of the hanger 1a' under the asymmetric parallel double-hanger system. In both two hanger systems, the hanger sudden fracturing at the end anchorage will lead to an obvious increase of stress in other hangers at a vicinity of local damage, as a loading impact was applied.

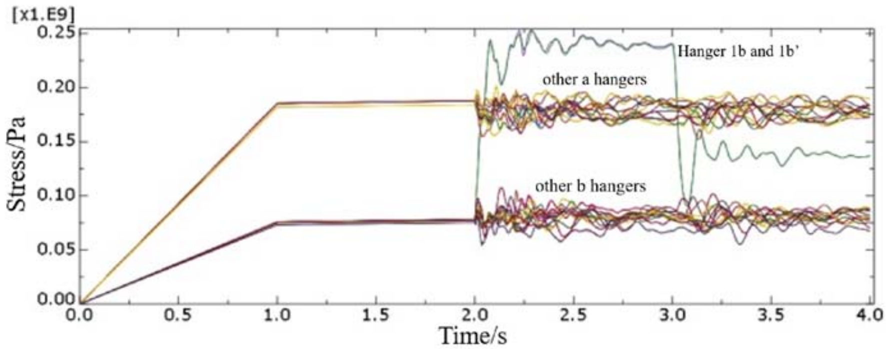
4.3 Maximum stress of remaining hangers after two short hangers continuously fracturing

After the sudden fracturing of short hangers (hanger 1 or hanger 1a) near the end of arch rib for two hanger design systems, the maximum tensile stress can be observed in hanger 2 in single hanger system or hanger 1a’ in parallel double-hanger system, which suggests these two hangers would be the next broken hanger for each case. As a result, the analysis of maximum tensile stress of remaining hangers should be divided into two parts, one with hanger 1 and 2 fracturing continuously in single hanger system, the other with hanger 1a and 1a’ fracturing continuously in double-hanger system.

Figure 7 shows the tensile stress variation of remaining hangers for the single hanger system (Fig. 7a) and the asymmetrical parallel double-hanger system (Fig. 7b) ,which is influenced by sudden continuous fracturing of two short hangers near the end of arch rib.



a) Single-hanger system (Case 1)



b) Asymmetrical parallel double-hanger system (Case 2)

Fig.7 Maximum stress of remaining hangers under continuous fracturing of two hangers

It can be seen from Figure 7 that the maximum tensile stress is 459 MPa in hanger 3 for the single hanger system, and 253 MPa in hanger 1b for the parallel double-hanger system. The tensile stress in hanger 1b is relatively small and beneficial to the safety of the residual structure.

As a result, if the tied-arch bridge is redesigned with the asymmetrical parallel double hanger system, the residual structure can still work with enough structural safety, in the case of failure and safety hanger at the same end anchorage fracturing continuously. The fact shows that a tied-arch bridge with the asymmetrical parallel double-hanger system will become a robust structure, when following a sudden fracturing of one or more short hangers. Instead, compared with the new design approach discussed in the paper, the residual structure with the single hanger system has less safety, because hanger 3 will be most likely to be the third broken hanger. Therefore this fact indicates that the remaining hangers may fracture continuously in a tied arch bridge with the single hanger system, which most likely will lead to the progress failure of the whole bridge.

5. Discussion and conclusions

In order to enhance tied-arch bridge robustness and avoid subsequent collapse due to hangers' local damage, a practical and novel design concept, named as the asymmetric parallel double-hanger system, has been proposed and evaluated in this paper. The asymmetric parallel double-hanger system is designed with one failure hanger and another safety hanger at each deck suspension point. The feasibility of this new design concept has been further evaluated and demonstrated by authors through the fatigue life analysis and dynamic time-history analysis of a case study, supported by a finite element model.

According to the fatigue life analysis, which is based on Miner linear cumulative damage law, the fatigue lives of two hangers are various due to the distinct stress amplitude inside. Therefore, the failure hanger, with higher stress, loses bearing capacity first, instead of fracturing simultaneously with safety hanger. Moreover, a dynamic time-history analysis has been conducted to simulate the transitory loading fracture impact due to one or more hangers fracturing.

A numerical model of the full-scale tied-arch bridge was also employed to compare the performance of proposed new double-hanger design system with the traditional one. Based on the results, it can be confirmed that the stress inside the safety hangers along the bridge have slight variations if one or two short failure hangers are broken, which subsequently can be the safety assurance for the rest of the structure. On the contrary, the bridge with traditional single-hanger system is more likely to experience further continuous fracture, thus triggering a whole bridge collapsing, when compared with proposed parallel double-hanger system.

In short, the robustness of tied arch bridge can be highly enhanced by implement the asymmetric parallel double-hanger system. The feasibility of developed double-hanger system has also been demonstrated by the alternative load path theory in the paper. In order to keep its perform ability, further analysis would be made in detail available.

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